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DISCUSSION OF UPLIFT PRESSURES IN CONCRETE DAMS

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DISCUSSION

Ross M. Riegel,[5] M. ASCE.—A composite diagram of foundation uplift observations in four dams of the Tennessee Valley Authority (TVA) is shown in Fig. 17. The dams with their respective maximum heights are as follows: Fontana, 480 ft; Hiwassee, 307 ft; Cherokee, 202 ft; and Douglas, 175 ft. This composite is made from measurements upon fourteen sections in the four structures. The measurements were made by methods similar to those described by Mr. Keener.

In each dam a thorough job of cutoff grouting has been done. Drains downstream from the cutoff have been provided at intervals ranging from 7.2 ft to 10 ft on centers, the average being about 8.25 ft. The sections entering into the composite were taken at the highest reservoir stages available, but tailwater elevations were moderate. Nothing is available at high stages of tailwater.

The solid line in Fig. 17 represents composite readings of intensity in the area of contact or within the top 2 ft to 3 ft of foundation. Heavy dotted lines are shown extending to a point representing the composite position of drainage outlets in a gallery. The drain outlets do not coincide as a rule with the measuring sections. There is some justification for the view that the pressure gradient should be no higher than the drain outlets, and the heavy dotted lines represent a gradient corresponding to this view.

The composite is plotted with the same scale system as that used by Mr. Keener and, therefore, it is easy to transfer the average curve in Fig. 10 to that of the eight dams in Fig. 17. Study of Fig. 10 and reference to the author's description of the Hoover Dam (Arizona-Nevada) measurements indicate the following: If the most recent measurements at Hoover Dam (after the later drilling and grouting) were used, and if Owyhee Dam, in Oregon (which seems to be something of a "freak"), were omitted, the two average diagrams would be much closer together. The two curves show the marked effect of combined cutoff grouting and drainage.

Fig. 17 also shows an intensity assumed for design in the four TVA dams. Actually the gradient used terminates at maximum assumed tailwater elevation, but, of course, no observations have been made for such conditions. The gradient shown merely indicates the comparison between the observed data and a design gradient referred to the same tailwater.

Data on Norris Dam in Tennessee are not included in the TVA composite. This dam was designed by the Bureau of Reclamation (USBR) of the Department of the Interior but was built by the TVA. The original design called for a grouted cutoff and for so-called observation holes downstream from the cutoff into a drainage gallery at intervals of 50 ft. Other drain holes were added during construction and numerous drain holes were drilled in the

Note.—This paper by Kenneth B. Keener was published in June, 1950, as *Proceedings—Separate No. 25*. The numbering of footnotes, equations, tables, and illustrations in this Separate is a continuation of the consecutive numbering used in the original paper.

[5] Head Civ. Engr., Design Dept., TVA, Knoxville, Tenn.

spillway apron. Observations on uplift cells showed reduction of uplift pressure as the additional holes were drilled. At present the uplift pressure diagram closely resembles the average diagram plotted by Mr. Keener. Probably an important factor is the extensive program of "consolidation" grouting which was applied to the general foundation of the dam in addition to the cutoff grouting.

The author has referred to tests conducted by closing drain holes temporarily. In June, 1936, a test was made at Norris Dam by closing all the drains for 24 hr. During this period increases of 30% were observed at cells 18 ft and 35 ft from 1 drain hole and lesser effects were observed at other

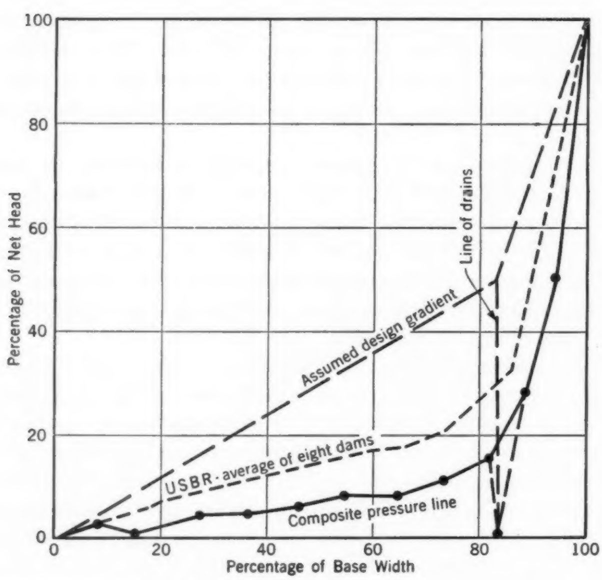

Fig. 17.—Composite Foundation Uplift Intensity for Four Dams of the Tennessee Valley Authority

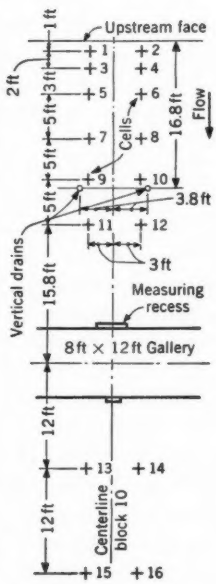

Fig. 18.—Plan of Location of Pressure Cells, Hiwassee Dam in North Carolina

cells. At Hiwassee Dam in North Carolina tests were made on different days on four separate blocks in which uplift cells had been established by closing all the drain holes in each block. The plugs were removed as soon as noticeable increases in pressure were observed. This took place at intervals of from 2 hours to 22 hours in the respective blocks.

Measurements of Pore Pressure in Concrete.—The author has described certain attempts to measure pore pressure at Hoover Dam. Similar attempts were made at Norris Dam. In each of three blocks of this dam six cells, for the observation of pore pressure, were installed on lift joints at El. 875 to El. 880, which is at a depth of 145 ft below the spillway crest, and where the external static pressure would have a maximum value of 63 lb per sq in. with lesser

values at lower reservoir elevation. These cells consisted of porous concrete blocks 12 in. square, each of which was connected with an observation point in a gallery by two small pipes. During a period of four years no pressures were ever observed on sixteen of these eighteen cells. In one cell, in block 37, at a distance of 2.5 ft from the water surface, a maximum pressure of 13 lb per sq in. was observed in June, 1936, which was the month in which the reservoir was filled for the first time. Subsequently, lower pressures were noticed ranging from 0 lb per sq in. to 8 lb per sq in. during the four-year observation period. The observer concluded that a small crack connected this cell with the water surface and that this crack subsequently closed up—probably from a swelling effect at the surface of the concrete.

In another cell in block 43, at a distance of 48 ft from the surface, a maximum pressure of 13.5 lb per sq in. was observed in May, 1939. In this cell the water flowed from the connecting pipes at times, whereas at other times there was a flow toward the cell and water had to be supplied to the pipes to keep them full. It was concluded that a crack connected this cell with a contraction joint which was known to contain water under variable pressure conditions.

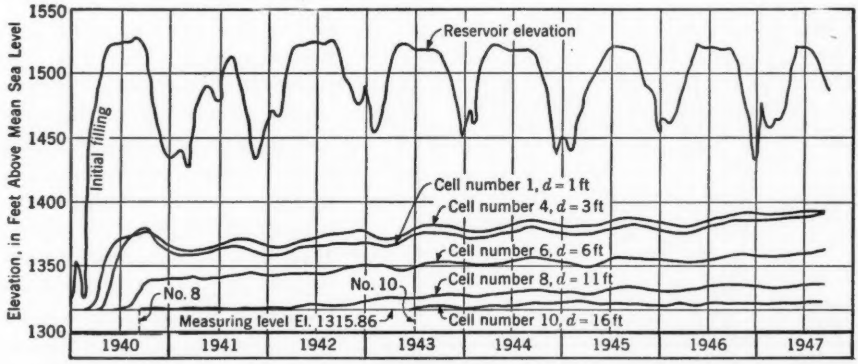

Fig. 19.—Record of Pore Pressure Measurements, Hiwassee Dam in North Carolina

In another block at Norris Dam a set of six sacks of gravel was placed at locations between the lift joints in a manner similar to that used at USBR's Gibson Dam (in Montana), and no pressure was ever recorded from these sacks. In short, no definite values of pore pressure were obtained in this dam during the four-year period of observation.

On Hiwassee Dam, however, some very definite results were obtained. Fig. 18 shows the location of a number of cells placed at one level in the middle of a lift. These cells were developed by the USBR primarily for measuring pore pressures in earth. They were connected by an insulated wire inside a metallic tube to a measuring station, and their principle of operation was similar to that of the Goldbeck cell. The application of a carefully controlled air pressure will cause an electric circuit to be broken when internal and external pressures on the diaphragm in the cell balance.

The record of these cells is shown,[6] for a seven-year period in Fig. 19. The depths d indicate the relative levels at which the cells were placed. Only the higher of the two pressures measured at each distance is recorded. They show a consistent pattern of slowly rising pore pressures. Fig. 18 shows the relation of vertical drainage wells in the concrete to the measuring cells. No appreciable pressures have yet been detected downstream from these drainage wells.

At the Fontana project in North Carolina another installation was made as shown in Fig. 20. The small square symbols indicate the location of the same kind of pore pressure cells used on the preceding projects, whereas the cells marked PT were the so-called piezometer tips, also obtained from the USBR. This tip is merely a small chamber protected by a porous stone and connected by two tubes with an observing point. The use of the two tubes permits both of them to be filled with water, after which one is closed by a valve and the other is permanently connected to a Bourdon pressure gage. It was expected that the piezometer tips would record higher pressures and that the other cells would record lower pressures in a manner similar to those obtained at Hiwassee Dam. However, the concrete at Fontana Dam, at least in the block observed, appeared to be more permeable than that at Hiwassee Dam. The pore pressure cells showed a rather rapid rise and then began to fall—apparently from overstress. The USBR stated that these cells were to be limited to 200 ft of water pressure and this limit seems to be somewhat on the high side.

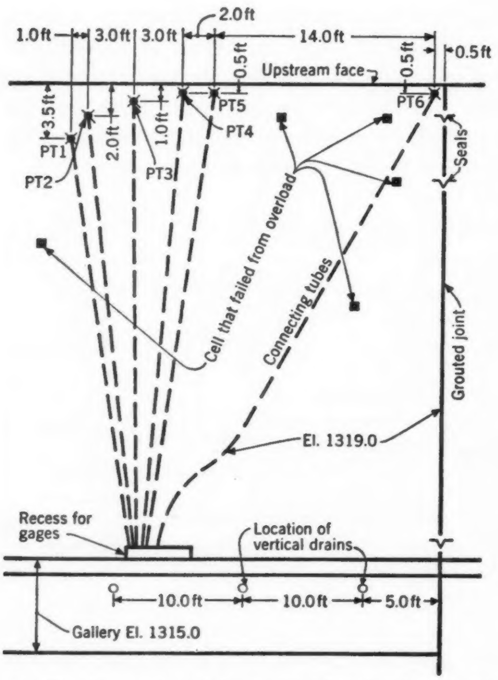

FIG. 20.—LOCATION OF PIEZOMETER TIPS, FONTANA DAM IN NORTH CAROLINA

The piezometer tips, however, afforded the record shown in Fig. 21, which is reasonably consistent. The record marked CT is from a Carlson-Terzaghi cell similar to that referred to by the author, which was installed in another block of the dam but at the same elevation as the piezometer tips. The measuring disk of this meter was at a depth of 18 in. from the surface. The PT installation, however, encountered trouble. The tips should have been connected with the gages by copper tubing, but, since the installation was made

[6] "Methods and Instruments for the Measurement of Performance of Concrete Dams of the TVA," by C. E. Blee and R. M. Riegel, *Proceedings*, Conference on Large Dams, Stockholm, Sweden, June, 1948, p. 5 (R45), Fig. 1.

during wartime, the copper tubing could not be obtained and a plastic tubing was substituted. This tubing had to be bent outside the concrete to make satisfactory connection with the gages. After about eighteen months, the period shown on the record, the plastic tubes began to fail from the combined effects of flexure and pressure. Repairs were finally made on five of the six cells by substituting copper tubing. However, about one year of record was lost while these repairs were made. Thereafter, the pattern of pressures was about the same as indicated in Fig. 21, except that the Carlson-Terzaghi cell showed about 95% of the external hydraulic pressure whereas the highest piezometer tip reading showed only about 90%. Pressures at greater depth are continuing to rise but very slowly.

The history of these measurements indicates some of the difficulties that must be anticipated in making measurements of this kind; and such investigations are much needed. On the three dams, TVA engineers have one quite satisfactory record, one failure, and one partly successful record—the over-all efficiency being about 50%. It can be stated, however, that these measurements have demonstrated that pore pressures of material intensity do exist in

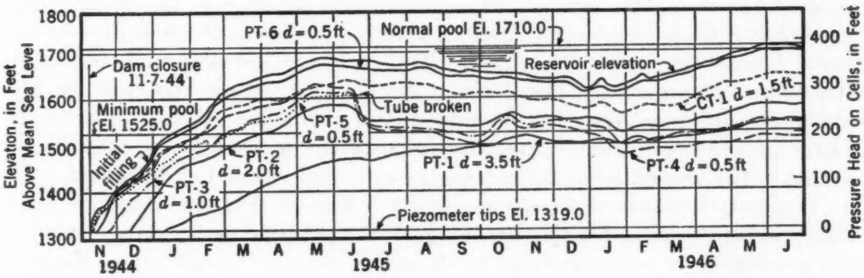

FIG. 21—RECORD OF PORE PRESSURE MEASUREMENTS, FONTANA DAM IN NORTH CAROLINA

the region adjacent to the reservoir water and that a gradient is apparent between the reservoir and drainage openings or galleries. Whether there is any observable pressure downstream from the drainage wells is not known.

The foregoing data are intended to amplify and confirm the data submitted by the author and it is believed that they have considerable value from this standpoint.

WILLIAM P. CREAGER,[7] HON. M. ASCE.—A valuable addition to the existing data regarding uplift pressures under masonry dams founded on rock has been presented by Mr. Keener. This paper, together with data offered by Ivan E. Houk,[4] M. ASCE, and others, assists materially in estimating the pressures to be expected under such structures.

The author has stated (under the heading "Uplift Design Assumptions") that the

> "* * * most common uplift design assumption used by the USBR [Bureau of Reclamation] in the past * * * is that uplift pressure on the

[7] Cons. Engr., Buffalo, N. Y.

[4] "Measurements at Existing Structures," by Ivan E. Houk, *Civil Engineering*, September, 1932, p. 578.

base varies uniformly from full-reservoir pressure at the upstream toe to tailwater elevation or zero, as the case may be, at the downstream toe and that the pressure acts over two thirds of the base area."

The writer believes that there is now sufficient evidence that uplift pressure is exerted over 100% of the base or at least close enough to that value to use for all practical purposes. Under this assumption, the uplift pressure may be divided into two parts:[8,9]

1. Tailwater pressure over the entire area of the base, plus
2. A pressure equal to the net head on the dam at the upstream toe, varying uniformly to a percentage, p, of the net head at the grouted cutoff and drains and thence varying uniformly to zero at the downstream toe.

For practical purposes, the uplift pressure may be considered to be equal to the tailwater pressure plus a percentage, p, of the net head on the dam at the upstream toe, varying uniformly to tailwater pressure at the downstream toe, all exerted over 100% of the area.

The percentage, p, has a value which depends on the effectiveness of the grouted cutoff and the drains. It is obviously a function of the relative permeability of the grouted cutoff and the drains, and the condition of the foundations downstream from the cutoff. This value is not known beforehand. However, the previously mentioned tests on seven dams reported by Mr. Houk, six additional tests by the author, and about five others known to the writer indicate conclusively that the value of the percentage, p, may be expected not to exceed 50%, if the foundation is well grouted and drained.

Incidentally, since the tailwater uplift is known, Fig. 10 would be more enlightening if it recorded the percentage of the net head on the dam instead of the percentage of reservoir head to show the aforementioned second division of uplift pressure.

A. Warren Simonds,[10] M. ASCE.—An excellent summary of the experiences of the USBR in observing uplift pressures beneath concrete dams is contained in this paper. The installation of measuring equipment, method of making observations, and results of the measurements are presented. For design purposes, the observed uplift pressures at the various dams are expressed as averages plotted in percentage of reservoir head against percentage of base width. This tends to reduce the magnitude and distribution of pressures beneath the dams to a common base for comparison.

In using the graphs shown in Fig. 10 for design purposes, it should be remembered that the plotted results are average values and therefore will not necessarily furnish an exact comparison for an entire foundation area at some other site. Foundations vary widely; geological irregularities or defects may make a large difference over a local area in the resulting uplift pressures due to full reservoir head. Furthermore the uplift pressures vary from time to time. The magnitudes of the pressures fluctuate normally with the variations in

[8] *Transactions*, ASCE, Vol. 111, 1946, p. 1202.
[9] *Ibid.*, Vol. 114, 1949, p. 215.
[10] Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

reservoir water surface after an appropriate lag in time which is dependent on the degree of permeability of the foundation.

The case history of the uplift pressure at Hoover Dam in Arizona-Nevada (which the author describes briefly) is of unusual interest because of its relation to certain physical and geological conditions encountered in that foundation. The uplift pressure at this dam was caused by water from two different sources—cold water from the normal percolation of reservoir water into the foundation, and warm alkaline water coming from springs which apparently originated at a considerable depth beneath the surface.

During the excavation of the foundation area, several warm springs were encountered along the Nevada side of the canyon. These springs were opened by drilling and pipe connections were made to them. The pipes were led into one of the lower galleries of the dam for grouting purposes after a sufficient cover of concrete in the dam had been placed over the areas from which the springs emerged. Other physical defects, in addition to the usual joints and cracks in the foundation rock, were several shear zones in the right central part of the foundation area. Some of these shear zones were in close proximity to the warm alkaline springs which occurred beneath the uplift pressure pipes in line B, Fig. 11.

After sufficient concrete had been placed above the warm spring area, an attempt was made to grout the springs by pumping neat cement grout through the pipes previously installed for this purpose. When the cement grout came in contact with the warm alakline water, a flash set of cement occurred before the springs could be grouted. This resulted in plugging the pipes that provided the only means of drainage from the warm spring area. The warm water subsequently found its way into the shear zones in the foundation area and caused the high uplift pressure in 1938 which was indicated by the installation along line B, Fig. 11. Subsequent grouting operations using special cements and retarders, and the drilling of foundation drains in strategic places reduced the magnitude of the uplift pressure until it was no longer excessive. It is interesting to note that on the Arizona side of the canyon, where there were no warm springs or shear zones, the uplift pressure along line C, Fig. 11, was much less than that which occurred along line B.

It may be well to emphasize that uplift pressure patterns beneath concrete dams do not vary as a straight line from the upstream face to the downstream face of the structure. All the patterns of observed pressures show a range from full reservoir pressure at the upstream face to a marked reduction at or near the line of foundation drains. From the line of drains to the downstream face, there are some irregularities, but, under normal conditions, the maximum pressures in this area are fairly small.

Uplift pressures beneath concrete dams can be controlled to a large extent by a reasonable program of grouting and drainage. Adequate grouting will minimize the flow of water beneath the structure and drainage will prevent the development of high pressures. During the construction period of a concrete dam, it happens frequently that some areas of the foundation receive inadequate treatment. It is only normal, therefore, to supplement the original foundation treatment by additional grouting and drainage after the reservoir has filled.

The need for such additional treatment is indicated by the uplift pressures observed at the base of the dam. For this reason the observations for uplift pressure serve as a valuable indicator of conditions in the foundation beneath a dam after the reservoir has filled.

Fairfax D. Kirn[11].—Valuable information regarding uplift pressures measured at some of the USBR dams now in service is presented in this paper. These measurements clearly show the effect of adequate drainage systems in reducing the uplift pressure at the line of drains. Foundation grouting was also shown to be a useful medium for relieving unduly large uplift pressures. The measured uplift pressures do not, of course, give any information regarding the extent of the area over which the uplift acts.

Under the heading "Uplift Design Assumptions," Mr. Keener states:

> "The maximum and most common uplift design assumption used by the USBR in the past for concrete gravity dams is that uplift pressure on the base varies uniformly from full-reservoir pressure at the upstream toe to tailwater elevation or zero, as the case may be, at the downstream toe and that the pressure acts over two thirds of the base area."

Diagrams determined from measured uplift pressures under dams in service as given in Mr. Keener's paper show that the average of the observed pressures is generally well under the foregoing design assumption. Based on these observed measurements and also on the knowledge that excessive uplift pressures can be prevented, or controlled if they occur (by an adequate drainage system and foundation grouting), it would seem logical to assume that the uplift pressure varies uniformly from full reservoir head at the upstream face of the dam to some percentage of the difference between reservoir head and tailwater head plus tailwater head at the center line of drains, and then from there uniformly to tailwater head at the downstream face. The area over which the uplift pressure acts is somewhat of a guess. The trend at the present time is to assume that the uplift pressure acts over 100% of the area of the base. However, regardless of published laboratory tests (which are too few in number to be conclusive) and hypothetical theories and until more exact methods are developed for determining the area over which uplift pressures act, the writer can not subscribe to the 100% area assumption. As long as cohesion exists, it would seem impossible that uplift pressure can act over 100% of the area of contact whether this area be considered to be in the concrete, the rock, or the bond between the concrete and rock.

In computing safety factors for transverse cross sections of gravity dams, it should be noted from Eqs. 1 and 2 that high stability is indicated by a high shear friction factor but by a low sliding factor. Eq. 1 is the generally accepted sliding factor equation. It is believed, however, that a more satisfactory equation for computing the stability of the dam against failure by sliding can be obtained directly from Eq. 2. If the cohesion (unit shear resistance) is assumed to be zero in Eq. 2, it reduces to

$$f_r = \frac{(W - U)\,\phi}{H} \qquad (3)$$

[11] Head, Trial Load Group, Dams Div., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

Eq. 3 takes into account the coefficient of internal friction, ϕ, of the concrete and rock. When f_r, computed from Eq. 3, is greater than 1, safety against failure by sliding is indicated the same way as for Eq. 2. Eqs. 1 and 3, however, should rarely, if ever, be used for determining the stability of a gravity dam against failure by sliding, since the cohesion will always have some value if the rock at the dam site is at all suitable for a gravity structure.

In discussing the safety factor equations, Mr. Keener states (under the heading "General"):

> "Noting the position of U, the "uplift" in Eqs. 1 and 2, one can readily determine the effect of that item in influencing the final cross section of the dam. If the assumed uplift is higher than necessary, it will be reflected in an unduly large cross section and a more costly dam. Conversely, if the assumed uplift is too low, undesirable encroachemnt will be made on the normally used values of the factors of safety."

In addition to the uplift, Eq. 2 contains two other terms which also influence the final cross section of the dam and for which values must be assumed at the present time. These are the coefficients of internal friction, ϕ, and of cohesion, τ. Both terms, of course affect the magnitude of the shear friction factor. The cohesion, in particular, has a marked influence, since it is multiplied by the area, A_h, of the contact plane. Laboratory and field tests made to determine actual values for these terms would eliminate the necessity of using assumed values in Eq. 2 with the consequent penalty of having to design the cross section of the dam for higher factors of safety. Investigational work of this nature is now (1950) being undertaken by the USBR. When sufficient information is available regarding the distribution of the uplift force and the area over which it acts, the magnitude of the cohesion, and the coefficient of internal friction for any particular concrete and foundation rock, it will then be possible to reduce the requirement that the shear friction factor be five, or greater, as mentioned by Mr. Keener. The results will be a more economically designed dam.

The writer believes that the gravity method of analysis should be used only to determine an approximate cross section for the dam. A gravity dam is a statically indeterminate structure the same as an arch dam, and any analysis that neglects effects of the transfer of load horizontally to the abutments by twist and beam action gives erroneous stresses and safety sactors. For substantiation of this statement one need only look at the diagonal cracking that occurs near the abutments of practically all gravity dams. A simple gravity analysis requires that a slice of the dam be analyzed as an isolated structure with no consideration of the effect of load transferred by horizontal elements to the canyon walls. Such an analysis gives stresses and safety factors against failure by sliding which are identical at the same elevation for every similar cross section in the dam regardless of its location in the structure. An analysis of the dam acting as a monolithic structure by the trial load method, however, discloses that this is not the case. The trial load analysis shows that safety factors at the inclined abutment planes of the shorter cantilever sections which base on the canyon walls are the critical ones and not those which occur at the bases of the higher sections in the central part of the dam.

If the term, sec ψ, is inserted in Eq. 2 and the area, A_h, replaced by the area, A, for the inclined base plane of the cantilever, the equation becomes

$$f_r = \frac{(W - U)\,\phi \sec \psi + A\,\tau}{H} \qquad (4)$$

in which ψ is the angle between the vertical and the inclined abutment plane.

Eq. 4 gives the shear friction factor at the inclined abutment plane for any cantilever section with its base on the canyon wall. Since sec ψ is never less than 1 and the area, A, is always greater than A_h, Eq. 4 gives a value for the shear friction factor on the inclined base plane of the cantilever which is greater than that computed for a horizontal plane at the same elevation from Eq. 2. This statement, however, is not true if the factor is computed for the load distribution determined from a trial load analysis of the structure. The transfer of load horizontally to the abutments by twist and beam action increases the magnitude of the denominator, H, in Eq. 4 to a greater extent than the numerator is increased, and the resulting shear friction factor is considerably less than the factor determined from the same equation for a gravity analysis.

SERGE LELIAVSKY BEY,[12] M. ASCE.—The author of this interesting paper deserves credit for presenting, in a concise and lucid style, a number of measurements of pore pressure observed at the dams of the USBR. Within its scope this paper has certainly achieved its purpose, and although the observations forming its subject matter are more in the nature of maintenance routine than pioneer work, they constitute a valuable contribution to the subject under discussion.

From another standpoint, also, the paper is important; that is, as a standard showing how such measurements must be observed and recorded. It is to be hoped that it will encourage other bodies in charge of dam maintenance to undertake and publish similar experiments.

The following points are suggested not as an adverse criticism, but as improvements that may possibly enhance its importance. In the first instance, the author seems to be rather reserved in stating his conclusions explicitly. The fact that they are negative—that no new point has been discovered—does not reduce their weight and value, particularly if opposed to the painstaking care with which these experiments were conducted. The diagrams in Fig. 10 are indeed very much the same as, or equivalent to, some published earlier.[4] The fact is rather instructive and deserves particular mention as a significant conclusion.

Another point calling for comment concerns the term "uplift," as used in the title and text of the paper. Modern dam design distinguishes between the uplift force, as it appears, for example, in Eqs. 1 and 2, and the pore pressure (or interstitial pressure) which is a hydraulic concept and is supposed to be investigated quantitatively by means of pipe measurements, such as those described in the paper. Admittedly, the two would be numerically equal if the effective area were assumed to be 100%, but the writer has shown, exper-

[12] Director, Designing Service, Projects Dept., Ministry of Public Works, Cairo, Egypt.

[4] "Measurements at Existing Structures," by Ivan E. Houk, *Civil Engineering*, September, 1932, p. 578.

imentally, that this assumption was not realized in nature; in fact, according to the results of his tests, the effective uplift area to be included in the calculation of a dam is 85%.[13] Attention may also be called to the fact that, as stated by the author under the heading "Uplift Design Assumptions," in a number of dams built by USBR, the effective area was taken to be only two thirds. The distinction between the two concepts in question (that is, hydraulic pore pressure and uplift force) is, however, much deeper than this arithmetical difference, for one of them is a scalar quantity whereas the other is a vector.

It follows, therefore, that as an insurance against possible misunderstandings, the title of the paper could have been altered into "Pore Pressures in Concrete Dams," with corresponding corrections in the text. This would also have the advantage of avoiding the feeling of disappointment on the reader's part when he discovers the subject of a paper is different from its title.

The third point involves Eqs. 1 and 2 used by the author in estimating the safety coefficient, as influenced by the uplift factor (see under the heading "General"). These formulas are not intimately correlated with the main subject of the paper but, nevertheless, call for comment as being somewhat reminiscent of the early period (middle nineteenth century) when structures, such as bridges and dams, were commonly designed from the principle of the breaking load. Total weight, total shearing force, and various other similar concepts were characteristic of the period. Criteria based on such "total" values and their counterparts, the "average" stress and safety factor, have since been replaced by design methods controlled by local stress intensities. From this standpoint the principle of returning back to the total values (as those in Eqs. 1 and 2) does not seem to be fully justified.

A. H. Davison,[14] M. ASCE.—For its presentation of factual data on experience with uplift pressures in concrete dams, as well as for the suggestion that such data are useful in considering uplift assumptions for proposed dams in order to avoid extravagant dam sections in design, this paper is a valuable addition to the literature.

Avoidance of extravagant dam sections in design may be attained by substituting a rational approach to the determination of uplift pressure for an assumed uplift pressure.[15]

A direct approach to the problem of determining uplift pressures involves, first, the computation of the hydraulic gradient through the dam and then the application of the corresponding uplift shown by the computed hydraulic gradient. Taking Fig. 8 as an example of factual data applicable to Marshall Ford Dam (in Texas), and applying the suggested procedure for comparison, the computed results check fairly closely with the factual data and indicate that the design assumption for uplift shown in Fig. 8 was unnecessarily conservative by about 10%.

Design considerations applicable to this example are:

[13] "Experiments on Effective Uplift Area in Gravity Dams," by Serge Leliavsky Bey, *Transactions* ASCE, Vol. 112, 1947, p. 444.

[14] Cons. Engr., Glen Falls, N. Y.

[15] *Transactions*, ASCE, Vol. 114, 1949, p. 239.

a. The section of dam considered is of homogeneous mass concrete, standing on rock foundation, and must be adequate to withstand the maximum design flood.

b. A design criterion for the adequacy of structure is that the resultant of overturning forces shall pass through the downstream limit of the middle third of the base of the dam. It is assumed that the concrete structure is set into the rock foundation sufficiently by construction methods to prevent sliding.

c. Natural foundation materials and materials of construction are elastic and compressible to a degree dependent on the material and on the amount and distribution of superimposed weight.

d. The Darcy law assumes that lost head through the structure increases uniformly along a straight line from the headwater to the tailwater in proportion to the distance along the line of creep of percolating water that would affect the hydraulic gradient. The assumption with respect to variation in the value of lost head is accepted as correct for the case of the theoretical rectangular wall, which represents uniform distribution of superimposed weight over any horizontal plane of the structure.

e. For other cross-sectional shapes than the theoretical rectangular wall, the variation of the lost head will be affected by: (1) The distribution of the weight of the structure across the width of the section, and (2) the variation in the position of the center of gravity of accumulated sectional area across the width of the section as denoted by coordinates measured from the source of hydrostatic pressure.

Fig. 22(*a*) has been drawn as a practical duplicate of Fig. 8, showing the typical dam section and adopting El. 486 as the average elevation of the base. Fig. 22(*b*) shows the theoretical rectangular wall constructed to the same critical dimensions as the practical dam section for the maximum design flood condition. Reference to Fig. 22(*b*) shows that the base width is 0.845 h to satisfy the design criterion for resistance to overturning, thus establishing the theoretical straight-line hydraulic gradient.

For a 1-ft length of homogeneous concrete gravity dam, l is the width and A is the area of progressive strips of the section, with x and y the corresponding coordinates of the center of gravity.

Friction head loss through the structure may be expressed by

$$h_f = K\,[w\,A\,(y + x)]^n \qquad (5)$$

in which h_f is the friction head, in feet; w is the weight of structure, in pounds per cubic foot; n is an exponent that varies with circumstances; and K is a dimensional coefficient that varies with circumstances. For a homogeneous structure like the concrete dam under discussion, Eq. 5 may become

$$h_f = K_1\,[A\,(y + x)]^n \qquad (6)$$

or as modified to apply it to this specific problem.

Tabulating the different elements for various values of l in Fig. 22(*b*) and satisfying the criterion that h_f must vary in a straight line from the reservoir

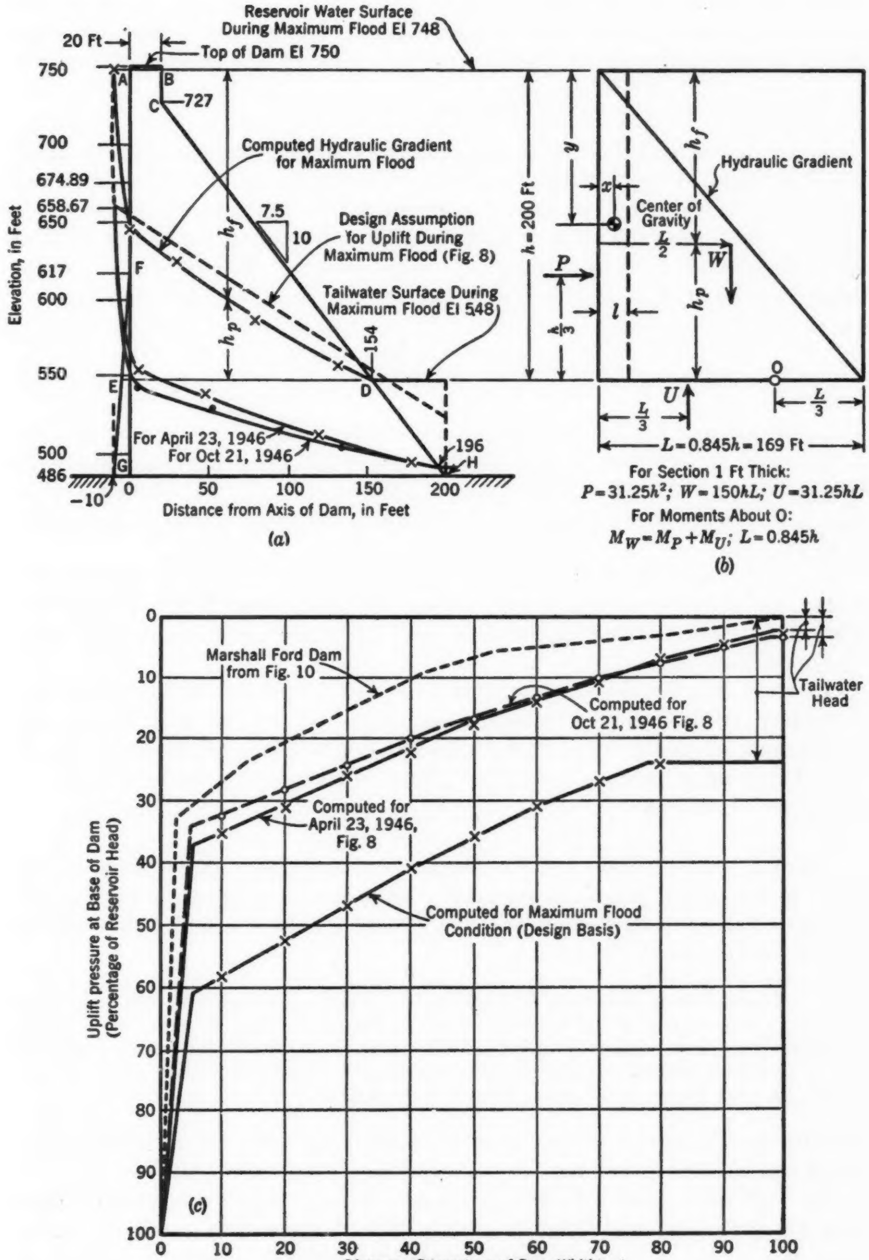
Reservoir Water Surface During Maximum Flood El 748
20 Ft
Top of Dam El 750
727
Computed Hydraulic Gradient for Maximum Flood
7.5
10
Design Assumption for Uplift During Maximum Flood (Fig. 8)
Tailwater Surface During Maximum Flood El 548
154
196
For April 23, 1946
For Oct 21, 1946
Elevation, in Feet
Distance from Axis of Dam, in Feet
(a)
h = 200 Ft
Hydraulic Gradient
Center of Gravity
L = 0.845h = 169 Ft
For Section 1 Ft Thick:
P = 31.25h²; W = 150hL; U = 31.25hL
For Moments About O:
M_W = M_P + M_U; L = 0.845h
(b)
Marshall Ford Dam from Fig. 10
Computed for Oct 21, 1946 Fig. 8
Computed for April 23, 1946, Fig. 8
Computed for Maximum Flood Condition (Design Basis)
Tailwater Head
Uplift pressure at Base of Dam (Percentage of Reservoir Head)
Distance (Percentage of Base Width)
(c)

Fig. 22

water surface to the tailwater surface, means that computed values of the compound factor $[A\,(y + x)]^n$ or some modification of that factor across the section must fall on a straight line. It is determined that the required factor is $\left[\frac{A\,(y + x)}{100{,}000} - 1.2\right]^{0.73}$ for this example.

The value of K_1 in Eq. 6 is established by considering that part of Fig. 22(a) which is bounded by the limiting elevations of the reservoir water surface and the tailwater surface for the maximum design flood. Tabulation of elements is made as before, and the value of K_1 is found by substituting the known value of 200 ft for h_f when l = 154 ft from the axis of the dam. The value of K_1 is thus determined as 16.9, and the formula applicable to Fig. 22(a) becomes

$$h_f = 16.9\left[\frac{A\,(y + x)}{100{,}000} - 1.2\right]^{0.73} \quad \text{(7)}$$

Application of Eq. 7 to the computation of the hydraulic gradient applicable to the total section area of Fig. 22(a) develops the three hydraulic gradients which conform to the three sets of factual data cited in Fig. 8. Also plotted for comparison is the "design assumption for uplift during maximum flood," taken from Fig. 8.

Fig. 22(c) shows a diagram of uplift pressure on a percentage basis similar to Fig. 10, reproducing the graph for Marshall Ford Dam. For comparison, the graphs corresponding to the computed hydraulic gradients in Fig. 22(a) have also been plotted.

Incidentally, the factual-data graph for April 23, 1946, in Fig. 8 appears somewhat erratic, possibly because insufficient time was allowed to establish equilibrium before the series of readings was taken. Possibly tailwater conditions, following or during a spring flood, had not become completely adjusted to headwater conditions.

Duff A. Abrams[16] M. ASCE.—No answers to the vital questions of hydrostatic uplift under large concrete dams have been presented in this paper. The only statement that can be called a general conclusion (under the heading "Uplift Design Assumptions") is:

> "Considering the extraordinarily low uplift pressures so far indicated over a short period for Shasta Dam, as shown in Fig. 9, and the average uplift pressures indicated by Fig. 10 for several dams over comparatively long periods, one is faced with the question of whether the uplift assumptions of the past have, in general, been too conservative."

Admittedly readings at Shasta Dam (in California) were extraordinarily low, but it will be shown that they prove something more than that.

The author is faced with the question of whether uplift assumptions have been too conservative. That is exactly the question that faced an earlier generation of engineers when they inaugurated USBR's extensive program of research on uplift in 1915.

[16] Cons. Engr., New York, N. Y.

The paper did bring out the disturbing fact that water under pressure was found at every point under all USBR concrete dams. Because of unscientific methods used, it is doubtful whether the full pressure of this water was determined in any instance. Examination of the data on uplift shows that the author misinterpreted the measurements and that the conclusions in the paper are erroneous and misleading.

Uplift Problems Not Solved.—Under the heading "Case History at Hoover Dam," the author states:

> "The essential case history of foundation uplift pressures at Hoover Dam is of interest. * * * Uplift pressures were first recorded in August, 1935, and have been continued to date. The higher pressures occurred during the first five years. Fig. 12, a running record for the holes in line B, reveals an increase in pressures until 1939 when remedial measures were undertaken by drilling more drain holes and by further foundation grouting."

In other words, Hoover Dam was no sooner in service than uplift became so alarming that search was begun for a "foundation treatment." After three or four years of "cut and try," grouting was adopted. This work was then so urgent that, in spite of shortages of labor and equipment, it was continued throughout World War II. Such episodes prove that uplift is real and that its problems have not been solved.

Averages of Averages.—The data supporting the paper consist of Fig. 10 and selected fragmentary uplift readings from five dams. Fig. 10 involves a bewildering array of averaging-out processes:

1. Averaging early semimonthly with later semiannual uplifts;
2. Averaging uplifts at three to seven different points in a given dam;
3. Averaging uplifts at pipes at widely different elevations;
4. Averaging readings over a wide range of reservoir heads;
5. Averaging uplifts measured by different methods;
6. Averaging uplifts before and after "foundation treatment";
7. Averaging all of the foregoing averages for a given dam;
8. Averaging of averages for small, medium, and large concrete dams.

This unscientific averaging is the principal basis for the conclusions in the paper. A low uplift in a small dam in Wyoming (Seminoe) cannot possibly offset the dangerous uplift under a large dam in Arizona-Nevada (Hoover), as implied by the average curve in Fig. 10. The author not only depends on the average curve but also draws it in disregard of the individual curves. At 20% of the base width the average uplift for eight dams is 22% of head; the average curve erroneously shows 27%.

At Hoover Dam on November 14, 1938, pipes B4 and C5 showed:

Elevations, in feet	B4	C5
Water in pipes	1,014	667
Tailwater	647	647
Uplift (feet of water)	367	20

Pipe B4 recorded eighteen and one half times as much uplift as pipe C5, but both pipes are about the same distance from the heel of the dam; hence they appear only as an average in Fig. 10. Pipe B4 revealed an uplift of 71% of the difference between lake and tailwater, or nearly three times the value assumed in design.

Fig. 10 shows that the downstream 60% of Hoover Dam had an average uplift from semimonthly, monthly, quarterly, and semiannual readings for fourteen years, equal to or greater than the "two-thirds full uplift" assumed in design. In October, 1938, measured uplift in pipe B6, 421 ft from the heel (Fig. 13) was 330 ft. The assumed uplift was 55 ft. In other words, with the lake only three-fourths full, the measured uplift was six times that assumed. All evidence suggests that, at full lake, the uplift at this point is about ten times that assumed in design.

For the first five years uplift under Hoover Dam was alarmingly high. It is stated that, for the next seven years, uplift was negligible; but in Fig. 10 the high and low values were averaged. The averaging-out processes conceal these dangerous conditions. The three instances cited may not represent the worst conditions at Hoover Dam.

TABLE 2.—EQUIVALENT ELEVATION OF WATER SURFACE IN PIPES

Pipe (Fig. 5)	Elevation (ft)	Δ (ft)	Δ^2
(1)	(2)	(3)	(4)
1A	310.0	−1.5	2.25
1B	310.8	−0.7	0.49
1C	312.8	1.3	1.69
1D	313.0	1.5	2.25
1E	312.0	0.5	0.25
2B	312.1	0.6	0.36
2C	310.9	−0.6	0.36
2D	310.5	−1.0	1.00
2F	310.7	−0.8	0.64
3B	315.0	3.5	12.25
3C	315.0	3.5	12.25
3D	309.0	−2.5	6.25
3E	310.0	−1.5	2.25
3F	309.0	−2.5	6.25
Mean	311.5	0.0	3.47

The most important consideration here is that structures (including large concrete dams) are not designed for average loads.

Even the period covered by Fig. 10 is open to considerable doubt. The paper states that the period covered was "* * * to the time of preparation of the chart * * *." One might guess that meant to the end of 1949. The only instance in which an independent clew can be applied shows that guess is erroneous. The average curve for the American Falls Dam in Idaho (Fig. 7(*a*)) is reproduced in Fig. 10, but it included no data later than 1939.

Friant Dam (*in California*).—This section is confined largely to lines 1, 2, 3 (Fig. 5). Line 4 is 65 ft higher and, therefore, contributed little to the uplift under the dam. A statistical analysis of the equivalent elevations of the water surfaces in pipes for fourteen of the sixteen pipes in lines 1, 2, and 3 (June 13, 1944), is given in Table 2. The mean elevation of the water in the pipes was 4.5 ft above tailwater. The author computes uplift as $4.5/212 = 2.1\%$ of head. The square-root-mean-squared difference is the standard deviation, $D = (3.47)^{\frac{1}{2}} = 1.9$ ft. The coefficient of variation, $V = 1.9/311.5 = 0.6\%$. This low coefficient is the first thing to arouse one's suspicions. The readings are the more suspect since water was discharged from the pipes. Fig. 5 is the USBR standard uplift form; it was described under the heading "Installations for Observation of Foundation Uplift":

"Three columns of the form are for records of pipes not discharging and four columns for pipe which, by reason of sufficient pressure, can discharge through the ½-in. outlet pipe connection."

Pipe 1A was entered in Fig. 5 as "not flowing," although water stood exactly at the top of the tee. The observer could not determine where to enter his record without first opening the stopcock. That dropped the pressure to 0; hence, the gages measured only small pressures built up during the few minutes required to prepare for readings.

As applied to concrete dams subject to uplift, two profound hydraulic principles are: (1) With a fountain running, the pressure at discharge is negligible; and, (2) with discharge cut off, the pressure rises instantly to that of the source. Uplift pipes are fountains designed for another purpose. In concrete dams the channels between the lake and the uplift pipes are more devious and constricted. The better the design and workmanship, the fewer and smaller the channels will be.

The entire treatment of the paper is based on two assumptions, first, that a large volume of water is required to produce uplift; and, second, that a large volume of water is always present at the base of a dam. Neither assumption is necessarily correct; in important instances both are erroneous. A film 0.00001 in. thick will transmit uplift. Movement toward an outlet would be extremely slow, since most of the head is lost as friction. Assume an initial flow of 11 in. per min at an orifice 4 in. wide at the bottom of an empty pipe. The discharge being 0.00044 cu in. per min, a period of 231/0.00044 = 525,000 min, or 1 year, will be required to discharge the first gallon. The rate would become slower as the pipe filled. Such a film can transmit dangerous uplift, but many years or centuries would be required to fill some of the long pipes; hence the pressure would never show on USBR gages. That is one reason why uplift was "slow in acting" in many instances.

The inside volume of a 2½-in. pipe is 0.4 gal per ft. Pipe B5 at Hoover Dam is 140 ft long and holds a barrel of water. Perhaps that explains why uplift always lagged behind (see Figs. 12, 13, and 14). Marshall Ford pipe 3Y is 40 ft long; Shasta pipe 4B is 65 ft long; both lagged behind the general trend. Small copper tubing should have been used. Cheaper and more elastic than steel, noncorroding copper tubing comes in rolls, can be strung anywhere without danger of leaks, and holds 1/600 as much water as a 2½-in. pipe.

In the opinion of the writer, Friant Dam readings are worthless as measures of uplift. Instead of uplifts of 1% to 3% of the head, all evidence indicates that, in the absence of bleeding of pipes, uplift would have been nearly 100% of the head over the entire base of the dam. The procedure at Friant Dam illustrates many of the unscientific features of USBR's uplift methods.

American Falls Dam.—The heavy zigzag lines in Fig. 7 show that the "average maximum uplift" was two and one half to four times the "average uplift." A comparison of the fourteen-year average with the "maximum point uplift" at Station 11 is given in Table 3. The maximum point uplift was 3.2 to ten times the fourteen-year average uplift. That is the true measure of the value of average curves in Fig. 10.

Marshall Ford Dam.—The author interprets Fig. 8 as proof of low uplift; but that is questionable in the face of the monotonous repetition of 495 ft and 496 ft, which appear fifty-seven times in 132 readings. Such wholesale coincidences in six pipes scattered over the base of a large dam do not represent

TABLE 3.—COMPARISON OF FOURTEEN-YEAR AVERAGE WITH THE MAXIMUM POINT UPLIFT

Description	Pipe Location[a]			
	12	32	52	73
14-yr average uplift (percentages from Fig. 7*a*)	18	6	3	2
Maximum point uplift (percentages from Fig. 7*b*)	83	19	19	20

[a] Pipe location as a percentage of the base width from the heel of the dam.

uplift. The answer is to be found in the fact that the gallery floor is at El. 493 ft. Bleeding of the pipes is the only possible explanation. Gage readings of 10 ft to 50 ft represent the small build-up of pressures before readings were completed. True uplift (lake full) may be any value up to $748-548=200$ ft of water over the entire base of the dam.

Shasta Dam.—"* * * the extraordinarily low uplift pressures * * * for Shasta Dam" were referred to at the beginning of this discussion. In 1947 the seventy-two monthly readings of six pipes 4D to 4I (Fig. 9), representing the downstream 350 ft of width of dam were:

Elevation, in feet	Number of occurrences
582	6
583	12
584	6
585	13
586	27
587	8

The mean elevation for the seventy-two readings is 585 ft; $D=1.5$ ft; and $V=1.5/585=0.3\%$.

Readings over the period of a year at six points under a large dam that show no response to changes of 47 ft and 39 ft in the lake, but come out with $V=0.3\%$, cannot be accepted as measures of uplift. It is as likely that two thirds of all trees in a forest will be of uniform height as measured by $V=0.3\%$. Some other explanation must be found.

As for Table 4, bleeding was practiced two years earlier on the same project; it was standard practice three years later. Analogy shows that Shasta pipes were bled before the gages were read. True uplift (lake full) may be anything up to $1065-585=480$ ft of water over the entire base, or ninety-two times what the author interpreted as "the extraordinarily low uplift pressures."

Hoover Dam.—In the preceding paragraphs attention has been called to the wide discrepancy between uplifts in two similarly located pipes in Hoover Dam, to the excessive pressures in pipes B4 and B6, and to the erroneous practice of averaging high uplifts before, with low uplifts after, foundation treatment.

The author points to Fig. 14 to support the conclusion (under the heading "Case History at Hoover Dam") that "By 1942 the pressures had been reduced

TABLE 4.—Comparison of Uplift Readings in Two Dams in the Central Valley of California

Dam	Year of observation	No. of pipes considered	No. of uplift readings	Coefficient of variation V (%)	Water in pipes above tailwater (ft)	Pipes were bled before readings
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Friant	1944	14	14	0.6	4.5	yes
Shasta	1947	6	72	0.3	5.2	not stated

to nominal amounts that have remained generally constant since about 1942." In July, 1938, and in July, 1947, lake elevations were essentially the same. Uplift readings at the seven pipes are presented in Table 5.

The author's inquiry stopped there with the conclusion that the drops of 91 ft to 280 ft represent a decrease from "drilling more drain holes and by further foundation grouting." The writer's conclusions are entirely different.

The five pipes with highest 1938 readings were essentially uniform in 1947, suggesting that they had been "trimmed" artificially. For pipes B1, B3, B4, B6, twenty-four of the forty-eight monthly readings in 1947 were 707 ft. Those "generally constant" values should have attracted the author's attention. From the standpoint of probability there is about 1 chance in 40,000 that such a condition could occur. Readings at Friant Dam and Marshall Ford Dam

TABLE 5.—Drop in Hoover Dam Uplift Readings in Nine Years

Date	Lake elevation, in feet	Water Elevation in Pipes, in Feet						
		1B	B1	B2	B3	B4	B5	B6
July 15, 1938......	1173	853	958	844	984	987	760	953
July 15, 1947......	1176	762	704	680	707	707	664	707
Drop............	....	91	254	164	277	280	96	246

pointed unerringly to the galleries. The paper identifies the Hoover gallery, where readings were made only as a powerhouse gallery. Official plans show the level floor of this gallery to be at 705.00 ft. The "generally constant" 707 ft is within a few millimeters of the exact elevation of the gage connections.

Bleeding of pipes is the only possible explanation. In 1942 Hoover Dam pipes were bled without reading gages. The determining influence of the gallery elevation on uplift readings at Hoover Dam is shown also by the following comparison of heights of water, in the pipes, above tailwater:

Dam	Year	Height, in feet
Friant	1944	$4\frac{1}{2}$
Marshall Ford	1946	6
Shasta	1947	5
Hoover	1947	60

The only reason that the Hoover Dam pipes did not show the same small height above tailwater as the three other dams is that the gallery is 55 ft too high. Instead of a "nominal amount," true uplift (lake full) may be anything up to $1229-647=582$ ft of water over the entire base of the dam.

The four pipes in the foregoing list define the uplift pattern for downstream as 80% of the dam. Little weight was given to pipes 1B, B2, and B5. Pipe 1B was read from another gallery and cannot be identified positively with a physical feature of the dam. Pipes B2 and B5 always lagged considerably behind (Fig. 12) and, hence, do not participate in the general pattern, probably because of leaks or other causes that the author should explain.

In the writer's opinion, there is no scientific evidence that the ten years of effort spent on supplemental grouting of the foundation had any effect in reducing uplift at Hoover Dam. Uplift was undoubtedly reduced somewhat by the repeated bleeding of pipes and additional drains, but chief reliance seems to have been placed on a systematic lowering of the lake elevation.

Bleeding of Pipes.—One version of this operation (as stated in the paragraph preceding Table 1) is:

> "Readings of the pressure gages having been made, the stopcocks are closed, the gages are removed, the stopcocks are opened, and measurement of the discharge is made in gallons, in cubic centimeters, or even in drops per minute."

After thus showing that bleeding of pipes was standard practice in 1950, the author states (under the heading "Installation for Observation of Foundation Uplift"):

> "As mentioned previously, after the pressure observations have been made, it has been the practice to remove the altitude gages and measure the discharge from the stopcocks. Naturally, the pressure dissipates for the time being and there is always the possibility that it may be fully restored before the next regular reading. Accordingly, the practice of estimating the amount of water discharged has been discontinued. It is doubtful if the revised procedure will make any material difference in the observations except in isolated cases."

No dams are named, and no dates are given. Bleeding of pipes is the dominating feature of USBR method. Hence the questions raised would have been most appropriate before the investigation had begun, but phrases such as "always the possibility," "may not be fully restored," "doubtful if," "any

material difference," and "except in isolated cases," are strange expressions at the end of a scientific report on a thirty-five year research program.

Concerning Hoover Dam in 1936, the author writes (under the heading, "Case History at Hoover Dam"):

> "* * * when the drains were closed intentionally, the uplift pressures increased and * * * when they were opened, there was some decrease in pressure."

Why, then, does the author in 1950 speculate as to the effect of opening uplift pipes?

At Hoover Dam April 15 to July 15, 1938, the lake rose for three months and then was stationary for three months. A brief study of the effect of this record on uplift is given in Table 6.

TABLE 6

Lake surface	Increase in Uplift, in Feet		
	Pipe 1B	Pipe B3	Pipe B6
Lake rose 72 ft in three months	17	45	38
Lake stationary for three months	117	25	24

Pipe 1B is nearest the heel, pipe B3 is nearest the middle, and pipe B6 is nearest the toe of the dam. As the lake rose 72 ft, water in pipes B3 and B6 rose two to three times as much as it did in pipe 1B. With the lake stationary, water in pipe 1B rose seven times as much as it did while the lake was rising 72 ft. There is evidence that uplift would have increased further had the lake been stationary longer. Bleeding of pipes would have destroyed all significant features of these tests. With those data in the books for eleven years it is difficult to understand either the author's speculations on delayed uplift action or the philosophy behind the practice of bleeding pipes while endeavoring to measure uplift.

Various phases of bleeding of uplift pipes are summarized as follows:

Year	Comment
1938	Hoover Dam (Fig. 13). No evidence of bleeding of pipes.
1940	Hoover Dam. Fig. 12 indicates that bleeding began coincident with "foundation treatment," and continued through 1947.
1942	Hoover Dam. Horizontal joint pipes bled without any reading of gages.
1944	Friant Dam (Fig. 5). Fourteen pipes gave a coefficient of variation V equal to 0.6%; the paper states that there was bleeding after the reading of pressure gages. Other evidence shows bleeding before a reading of the gages.
1946	Marshall Ford Dam (Fig. 8). El. 495 or El. 496 appear fifty-seven times in 132 readings of eleven pipes, and unerringly fix the elevation of the gallery; bleeding before reading the gages is the only possible explanation.

1947 Shasta Dam (Fig. 9). Seventy-two readings of six pipes gave a value of V equal to 0.3%; bleeding before reading the gages is the only possible explanation.

1947 Hoover Dam (Fig. 14). For four pipes, twenty-four of the forty-eight readings were 707 ft. This fact points unerringly to the elevation of the gallery floor; bleeding of pipes before reading is the only possible explanation.

1950 The paper gave four versions of USBR bleeding practices:
(*a*) Bleeding without reading gages (Hoover Dam, 1942).
(*b*) Bleeding before reading gages.
(*c*) Bleeding after reading gages.
(*d*) Bleeding of pipes has been discontinued.

Not only do the various versions in the paper conflict, but also the statement on which practice (*d*) is based is open to different interpretations. The writer interprets it to mean that bleeding has been discontinued; but the statement (paragraph preceding "Case History at Hoover Dam") was: "* * * the practice of estimating the amount of water discharged has been discontinued."

Repeated trials should not be necessary to convince an engineer that a pipe filled with compressed water will show discharge if a stopcock is opened. It is not clear what use has been made of discharge data. Whether the discharge is small or large does not influence uplift pressure. Bleeding converted the pipes into pressure-relieving drains.

Design Assumption.—There is no basis in the selected fragmentary data in the paper for either the uplift curves or the design assumption. The entire shape of the curves is dictated by two points—100% uplift at the heel and zero uplift at the toe—but both of those points are fictional. That is, 100 times 0 is the same as 0 times 100, and neither one contributes in the least to the uplift under the dam. The paper was made unnecessarily confusing by plotting uplift curves in Figs. 7 and 10 in the fourth quadrant and Figs. 8, 9, 13, and 14 in the first quadrant.

At Hoover Dam in 1938 (the only data free from evidence of bleeding), ten of the twelve readings of pipe B6 (nearest the toe) were higher than for pipe 1B (nearest the heel); in August, the difference was 107 ft. For 1936 to 1940 (before bleeding; see Fig. 12) the uplift near the toe was higher than near the heel. In other words, the first quadrant curves slope upward, whereas the author assumes a downward slope. With sufficient time, the uplift will be 100% of the head over the entire base of the dam, or three times what the author mistakenly calls two-thirds full uplift.

Origin of Uplift.—A dam that is nearly right is not good enough. Dams must be built safely or not at all. The designer cannot work intelligently until he knows what he is designing for or against. With concrete that is essentially impermeable, with several grout curtains, numerous drains, and an intimate bond between dam and rock, hydrostatic uplift that threatens to float the dam downstream is a decided anomaly. The paper provides no hint of the origins of uplift under USBR dams. Uplift cannot be treated as an isolated phenomenon. It is closely associated with: (1) Expansion of

concrete near the rock contact because of temperature rise from delayed hydration of too much of the wrong kind of cement; (2) horizontal movement of a dam due to the thrust of a lake; (3) vertical movement due to compression of concrete and rock; and (4) tilting due to a combination of items 1, 2, and 3. In the writer's opinion, cause (1) is the most important. All those topics have been under intensive study since the the beginning of Hoover Dam in 1933; none was mentioned in the author's paper under the heading "Case History at Hoover Dam."

KENNETH B. KEENER,[17] M. ASCE.—The observations, reported by Mr. Riegel, of the foundation uplift pressures under four high concrete dams constructed by the TVA are valuable in that they confirm the gereral results obtained by the USBR. By Fig. 17 it is seen that the average or composite pressure line for the TVA dams closely parallels that for the USBR dams although it is about 10% lower. Mr. Riegel's statement is unquestionably true: that the two lines would be closer together if only the measurements made at Hoover Dam, after supplemental drilling and grouting, were used and if the measurements at Owyhee Dam were omitted. One might go further and attempt to analyze the difference by the relative natural tightness of the foundations, by the degree of thoroughness in grouting the various foundations, or by a comparison of the foundation drainage systems. However, such a detailed study will not be undertaken, since it is sufficiently satisfying that Fig. 17 gives added reason for questioning "* * * whether the uplift assumptions of the past have, in general, been too conservative." (quoted from the paper under the heading "Uplift Design Assumptions"). Definitely, it appears that they have.

It seems reasonable to expect that some seepage will pass through the pores in the concrete between drains. With such an assumption the composite pressure line could not drop to zero where it crosses the line of drains. True, the pressure will be zero at any particular drain; but between drains it will be considerably higher, influenced by the size and spacing of the drain holes, by the water surface elevations of the headwater and tailwater, and by the permeability of the concrete and foundation rock. This is illustrated in Fig. 9: two rows of 3-in. drainage holes on 10-ft centers were drilled 50 ft deep from each of the drainage galleries in the region of pipes A and B, and yet the uplift pressures are considerably higher than the elevations of the galleries.

One is impressed with both the rapidity and effect on the uplift pressures obtained by closing drains at Norris and Hiwassee Dams. Those tests support the value of the continued operation of keeping all drains free and open.

The provisions for measuring pore pressure in the concrete of Norris Dam, similar to those employed at Gibson, Owyhee, and Hoover Dams, seem quite simple and should be conducive to direct results. Mr. Riegel reports that "* * * no definite values of pore pressure were obtained in this dam (Norris) during the four-year period of observation." His accounting of the methods and results of obtaining pore pressure seems to warrant a statement to the effect that at twenty-two different locations in the concrete of Norris Dam there

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was no pore pressure during the four-year period, and that pressures recorded at two other locations could be reasonably accounted for by cracks that led directly to definite water supplies or reservoirs.

Less simple means were used to determine the pore pressures in the concrete in Hiwassee and Fontana Dams. The seven-year record of results (Fig. 19) at Hiwassee Dam on cells near the upstream face is enlightening, as is the fact that no appreciable pressures were detected downstream from the drainage wells. The latter information is evidence of the usefulness of foundation drains in reducing pore pressures in the concrete in addition to their primary purpose of reducing foundation uplift pressure. Noticeable pore pressures are to be expected in concrete dams near their upstream faces, dependent naturally on the relative permeability of the concrete. The important question is "How far downstream do these pore pressures exist?"

The difficulties encountered with measuring equipment at Fontana Dam accentuate the problems involved in obtaining lengthy records of uplift pressures in concrete when somewhat complicated and delicate means are utilized.

Mr. Creager has raised the subject of area of intensity of uplift pressure. The writer did not intend to open this subject for discussion, since it has been so thoroughly covered by L. F. Harza,[18] M. ASCE. However, in completing the record of work done by the USBR on uplift pressures, a brief accounting was given of tests made in the USBR laboratories to determine the effective area over which uplift pressure acts. In addition, a factual statement (quoted by Mr. Creager) was made of the maximum and most common assumptions used by the USBR in the past. Both references justified Mr. Creager in discussing that much-debated subject.

The USBR has never made any measurements of uplift on its existing dams which would pretend to show the percentage of area over which uplift forces act. It is doubtful that a procedure for such measurements could be devised. The measurements (made continuously since 1926, of the intensity of uplift at various points under its concrete dams) cannot be analyzed to determine the effective area. Undoubtedly, there are many highly qualified engineers who frankly admit that they do not know closely the percentage of effective area and who await with technical interest the results of testing that will solve this vexing problem. If the theory had been proved conclusively, there would not be such advocating of testing to solve this problem.

Because of the data obtained in the measurement of uplift by the USBR and the TVA, the shape of the uplift pressure diagram given by Mr. Creager is a logical choice. Any assumed percentage of the product of the maximum theoretical intensity and the entire area should reflect not only the characteristics of the foundation rock and the concrete but also the locations and depths of foundation grouting and drainage systems. The uplift assumption then becomes a problem for each particular dam, to be solved by those who are familiar with the local information, with the construction designs, and with past experience under similar conditions.

[18] "The Significance of Pore Pressure in Hydraulic Structures," by L. F. Harza, *Transactions*, ASCE, Vol. 114, 1949, p. 193.

Mr. Leliavsky states that the writer "* * * seems to be rather reserved in stating his conclusions explicitly." The paper is largely a recital of numerous results obtained by the measurement of uplift under concrete dams. The data obtained augment those published seventeen years earlier. It would appear that the earlier data were not sufficient to convince designers that the uplift pressure diagrams could be modified and the total uplift reduced. Thus, for almost two decades the ultraconservative assumptions of the past were continued, as may be seen in Figs. 7, 8, 9, and 10. It was thought that the designer could quickly draw his own conclusions and the writer might appear presumptuous in even suggesting such apparent ones. Nevertheless, the writer did state (under the heading "Uplift Design Assumptions") that

> "* * * an uplift assumption more in keeping with the observations that have been made would result in smaller cross sections and appreciable reduction in cost."

The title "Pore Pressures in Concrete Dams," suggested by Mr. Leliavsky, would have been unacceptable because few data relative to the pore pressure within the mass of the concrete were presented in the paper. Most of the observations (and all those that were positive) were taken at the foundation contact, or 3 ft into rock.

Mr. Leliavsky's comment that "Criteria based on * * * the 'average' stress and safety factor, have since been replaced by design methods controlled by local stress intensities." arouses one's curiosity as to what degree that trend has become established. Such an application seems impracticable at the present time. Although additional stresses due to the structural action of the concrete mass of the dam may be predicted approximately, the haphazard stresses introduced by the exigencies of construction by variable local temperature conditions and by changed foundation topography encountered only after excavation would seem to preclude any but an extremely rough control of design methods by local stress intensities.

Mr. Kirn subscribes to a valuable use of the recorded uplift observations by modifying the assumed uplift pressure diagram from the former straight-line distribution to one that recognizes the effect of adequate drainage systems and foundation grouting. Mr. Kirn's opinion (that Eq. 1 and Eq. 3 should seldom, if ever, be used for determining the stability of a gravity dam against failure by sliding) is agreed with by the writer for the reasons so ably presented. The probability that, upon obtaining the additional information now being gathered, the shear friction factor may be reduced below five is encouraging; reduction (for example, to four) would reduce the cost of concrete dams appreciably.

Mr. Kirn's discussion on the analysis of concrete gravity dams, although not particularly pertinent to the subject of the paper, is a worthy addition to technical literature on the design of such dams. He has explained admirably and satisfactorily why a gravity dam is a statically indeterminate structure.

Mr. Davison has presented an interesting discussion of his theory that the shape of the pressure distribution curve through a dam is dependent predominantly upon the weight distribution. He ignores the effects of drain-

age systems and grout curtains in the foundation. It might be called to mind again that the observations reported by the writer were of pressures between the foundation rock and concrete and not of pressures throughout the mass concrete. To the writer's knowledge, the assumption that pressures vary according to local stress intensities has not been proved by any laboratory or theoretical evidence that would indicate permeabilities to be modified greatly by small changes in stresses at the small unit loads used in dam design. It would appear that such factors as the degree of hydration of the cement, the concrete-mix proportions, and the duration of saturation of the concrete would affect permeability more than local stress intensity.

Nevertheless, Mr. Davison's conclusion (by his method of computation) that the design assumption for uplift, shown in Fig. 8, was conservative only by about 10% should be comforting to the designers of Marshall Ford Dam.

The writer (and it is ventured to include all the many engineers connected with the uplift observation program) makes no claim to perfection in the investigation described, as the reader may observe from the factual data presented. There was no attempt to conceal facts. However, to have recorded all data collected in the past would have required a volume of such size as to be prohibitive.

Mr. Abrams is disturbed over the fact that water under pressure was found at every point under all USBR dams. To the writer this discovery is as enlightening as (and no more disturbing than) a much earlier discovery that water under pressure is found against the upstream faces of all dams. The main purpose of the paper was to show the magnitude of the water pressure (truly a normal condition) under concrete dams as well as the distribution of those pressures as reflected by actual measurements. Such information would be of value to designers of dams. The reader has the opportunity of judging whether or not the methods were scientific, for those methods were explained in considerable detail.

Because of Mr. Abrams' deduction that in all cases water was tapped off immediately prior to taking the uplift pressure readings, it appears that he doubts the value of the observations. A rereading of the paragraph regarding the manner in which the readings are made (see the text following Fig. 4) should clarify this question. Discharge from uplift pipe installations was measured after the uplift pressure readings had been made. It is true that in a certain particularly tight foundation, it was observed that pressures did not come up sufficiently at the end of 15 days to match readings that formerly had been made at 30-day intervals. Hence, as stated later in the paper, the practice of obtaining discharge from uplift pressure pipes was discontinued in all installations. This was to some extent an improvement in the methods, although entirely unnecessary at most installations.

Mr. Abrams is quite critical of Fig. 10. As stated in the paper, the drawing was prepared in a manner similar to Fig. 7, which in turn is typical of what can be shown graphically with data from uplift readings. Charts for all of the dams listed in Fig. 10 were made in a manner identical to Fig. 7. The individual plottings in Fig. 10 are the averages from charts similar to Fig. 7. The average, which may be termed the grand average, on Fig. 10 indicates a

general trend, and that is all it was intended to show. It may be seen from it that uplift pressure variation does not follow a straight line but is hyperbolic or exponential in form.

Mr. Abrams has questioned the use of iron pipe rather than small copper tubing for uplift equipment installations. It should be remembered, as stated in the paper, that holes are drilled through the uplift pipe into the foundation rock after initial grouting. If no pressure is shown at the gallery elevation and if pipes are conveniently situated, soundings are made by dropping a bell down the pipe. Both of those procedures would be extremely difficult, if not impossible, if copper tubing were utilized.

In closing his discussion on uplift at Hoover Dam, Mr. Abrams makes the bold prognostication that "With sufficient time, the uplift will be 100% of the head over the entire base of the dam * * * ." In the face of all tests and theory on the flow of liquids through permeable media, it is impossible to maintain that pressures must not drop in some manner from headwater to tailwater. They cannot at any time be 100% of the total pressure throughout the permeable medium.

Perhaps if Mr. Abrams had had the opportunity to read Mr. Simonds' able and clarifying discussion prior to preparing his own, his sweeping criticisms of the grouting and drainage program at Hoover Dam would have been modified. In any event Mr. Simonds, unknowingly, did answer many of them.

Mr. Simmonds' discussion is a valuable addition to the paper in that it shows the effect of local geological and construction conditions upon the development of uplift pressure at Hoover Dam. It also shows how excessive pressures may be reduced by corrective measures.

Much has been learned from the extensive program of observations of uplift under concrete dams. The discussions have contributed greatly to the practical and theoretical knowledge of this subject, so necessary to those who have the responsibility of designing dams that will be secure from failure and will be economical in construction. Laboratory testing or theoretical studies can never be as satisfying as actual observations on structures in service.

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